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Effect of Underground Beam on Seismic Damage of Railway Rigid Frame Viaduct

M. Onodera[1], K. Wada[2], A. Hino[3], Y, Murono[4]

[1] *Researcher, Railway Technical Research Institute, onodera.meguru.58@rtri.or.jp*
[2] *Assistant Senior Researcher, Railway Technical Research Institute, wada.kazunori.73@rtri.or.jp*
[3] *Researcher, Railway Technical Research Institute, hino.atsushi.44@rtri.or.jp*
[4] *Director, Railway Technical Research Institute, murono.yoshitaka.51@rtri.or.jp*

Abstract

In the 2016 Kumamoto Earthquake in Japan, it was found that the degree of damage to a railway rigid frame viaduct was different from to an adjacent one. Those structures were designed by the same 1991 design standards for railway structures in Japan. Those structures have met same demands with regard to running and structural safety under the Level 1 and Level 2 design earthquake motions. Despite of that, in one viaduct there was no damage, but in the other viaduct, cracks of piers occurred and took some days to restore.

There is structural difference in the presence or absence of underground beams between both structures. This difference seems to be a factor that caused a difference in damage degree. In this research, seismic response of those structures during the 2016 Kumamoto earthquake was investigated by numerical analysis. As a result, it was shown that natural period of a structure without underground beams was closer to the predominant period of the ground motion than that of the other structure with underground beams. The seismic response of a structure without underground beams increased due to the resonance. Furthermore, it was shown that the effect of the ground displacement on the structure without underground beams was larger than that with underground beams.

These results in this research indicates that it is possible to improve the seismic performance of structure with regard to the restorability by employing underground beams.

Keywords: Underground beam, Railway rigid frame viaduct, 2016 Kumamoto earthquake, ground displacement

1. Introduction

In the 2016 Kumamoto earthquake in Japan, large accelerations were recorded along the railroad. In the railway, Shinkansen vehicles were derailed[1] and noise barriers and girders of the rigid frame viaduct were damaged. On the other hand, the primary members such as columns and beams of the rigid frame viaduct remained relatively minor damage.

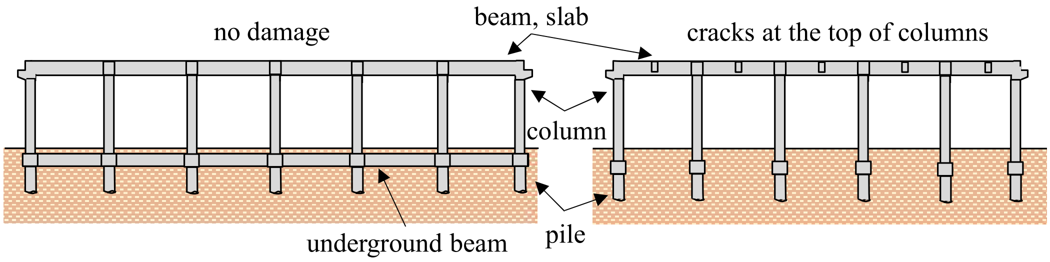
Among the structures that remained relatively minor damage, it was found that the degree of damage to a railway rigid frame viaduct was different from to an adjacent one. These structures were located at the same point and designed by the same 1991 design standards for railway structures in Japan[2]. Those structures have met same demands with regard to running and structural safety under the Level 1 and Level 2 design earthquake motions. Despite of that, in one viaduct there was no damage, but in the other viaduct, cracks of top of columns occurred and took some days to restore. As a result of comparing structural specifications of these structures, difference in the presence or absence of underground beams in the track direction was found out. Therefore, it was considered that the difference in the seismic behavior of these structures was caused due to this difference, and the degree of damage became different.

In this paper, an analytical evaluation of the effects of the presence or absence of underground beams on the behavior of railway rigid frame viaduct based on the damage cases in the 2016 Kumamoto earthquake was evaluated.

2. Analytical modeling

2.1 Target structures

The outline of the target structures are shown in Fig. 1 and the structural specifications of these structures are shown in Table. 1. Two railway rigid frame viaducts that are located at the same point and had difference in the degree of damage in the Kumamoto earthquake. These structures are designed according to the Design Standards for Railway Structures in 1999[2]. The structure with and without underground beams are described as "structure A" and "structure B".



(a) Structure A with undergound beams (b) Structure B without underground beams

Fig 1 — Outline of target structures

As shown in Table. 1, the difference in the structural specifications is not only the presence or absence of underground beams in the track direction, but also foundation types and the section area of columns. Structure A is cast-in-place pile and the section area of columns is 900mm×900mm. Structure B is steel pipe pile and the section of columns is 1300mm×1300mm. It seems that the reason why the section of columns of structure B is larger than that of structure A is the purpose to ensure the stiffness and capacity against for horizontal force.

Table 1 — structural specifications

Target structure	Structure A	Structure B
Number of Span	6	5
Height of structure	10.8m	11.8m
Length of span in a longitudinal direction	9.1m	8.7m
Length of span in a transverse direction	9.3m	4.8m
Section of column	0.9m×0.9m	1.3m×1.3m
Reinforcement ratio	0.78%	0.62%
Foundation type	Cast-in-place pile	Steel pipe pile
Diameter of pile	1.3m	1.3m
Length of pile	26m	31m

2.2 Modeling

A two-dimensional frame model was constructed for each structure with beam-spring elements for the direction of the track. The nonlinear characteristics of ground resistance of foundation was modeled according to the current design standards[3]. The modeling attributes are illustrated in Fig. 2. An "integrated model" in which structure system is connected to free ground system via soil-structure interaction springs was used. Thereby, the soil-structure interaction is expressed appropriately.

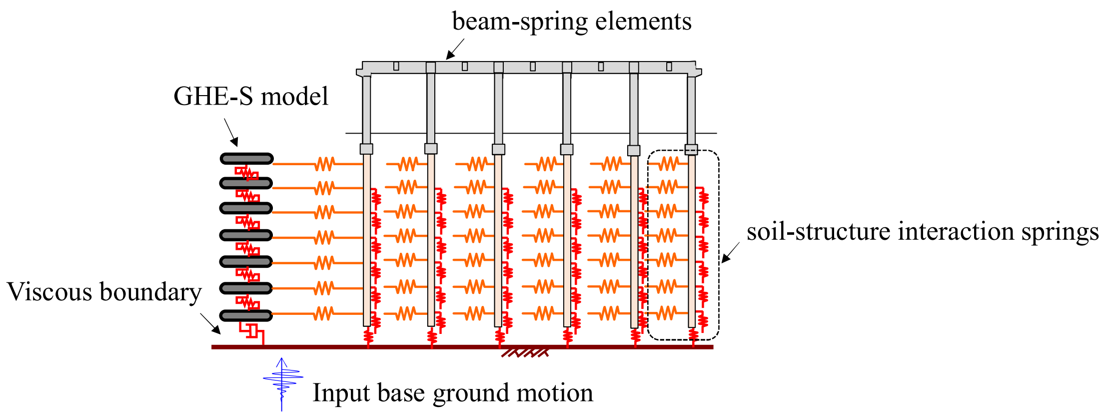


Fig. 2 — Analytical model

Table 2 — Ground conditions

Layer thickness (m)	Soil type	N-value	C (kN/m^2)	φ (deg)	γ (kN/m^3)	V_s (m/s)
2.7	clay 1	3	18	0	15	130
4.8	sand 1	5	0	26	17	137
2.0	clay 2	10	44	0	15	215
6.4	sand 2	19	0	33	18	213
4.0	clay 3	3	74	0	15	144
3.9	gravel 1	20	0	34	18	326
9.9	gravel 2	20	0	34	18	380

These structures is located adjacent to each other; therefore, ground conditions were set to the same as the each other. the ground conditions is shown in Table 2. The natural period of the ground is 0.58s. Nonlinear characteristics of the free ground were modeled by GHE-S model [4].

3. Vibration properties and Failure mode

A push-over analysis was performed on the constructed analysis models. In this analysis, the influence of the ground displacement on the structure is not taken into account by restraining the deformation of the free ground. The relationship between horizontal seismic coefficient and horizontal displacement at the top of structure is shown in Fig. 3. As shown in Fig. 3, failure mode of these structures are the same, the top of columns damage are progressed. The seismic coefficient at which crack occurs is 0.04 in structure A and 0.05 in structure B. In addition, the seismic coefficient of yielding is 0.50 in structure A and 0.53 in structure B. Thus, when the static seismic force that act upon these structures are the same, the degree of damage is almost equal.

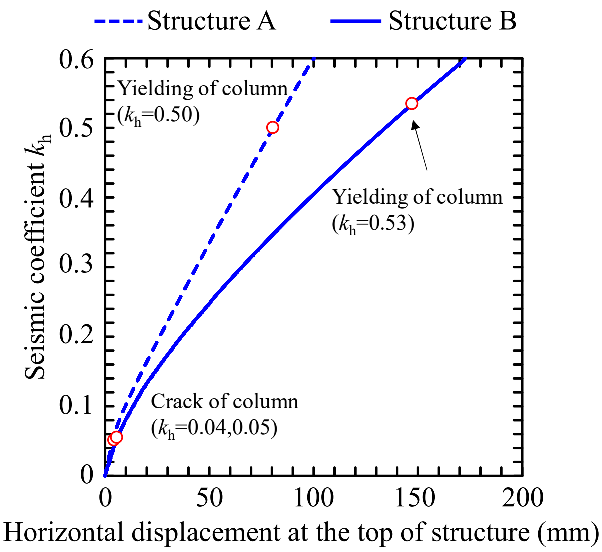


Fig. 3 — Horizontal seismic coefficient-displacement relationship

On the other hand, when the yielding of column occurs, the horizontal displacement is 81mm in structure A and 147mm in structure B, the horizontal displacement of Structure B without underground beam is greater than that of structure A with underground beams. This is because that the stiffness of the whole structure without underground beam is small. As a result, the natural period at when the initial yielding of column occurs is 0.8s (1.3Hz) in structure A and 1.0s (1.0Hz) in structure B. In addition, the primary natural period is 0.51s (2.0Hz) in structure A and 0.59s (1.7Hz) in structure B, and they are different.

As above, it was shown that the failure modes of these structures were the same and the horizontal seismic intensity at which each damage event occurred was almost equal, but the natural periods of both structures were different.

4. Comparison of seismic behavior under the 2016 Kumamoto Earthquake

4.1 Analytical Conditions

Dynamic response analysis was conducted for these structures. The base ground motion that calculated by using surface ground motion recorded in the 2016 Kumamoto earthquake was set to the input ground motion. The surface ground acceleration waveform and the ground conditions near the recorded point used for estimation of the input ground motion are shown in Fig.4 and Table. 3, and the acceleration waveform and

Fourier spectrum of input ground motion estimated are shown in Fig. 5. As noted in Fig. 5, the predominant period of the input ground motion is 1.0s.

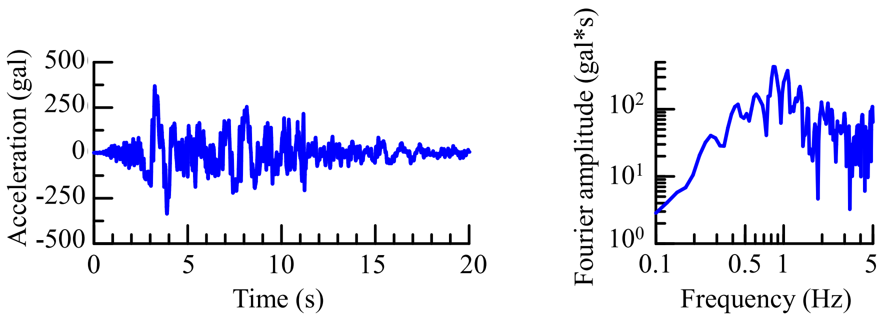


Fig. 4 — Recorded surface ground motion (2016 Kumamoto)

Table 3 — ground conditions near the recorded point

Depth (m)	**Soil type**	**N-value**	V_s **(m/s)**	**γ (kN/m³)**	**D50 (mm)**
0.0~3.2	clay	6	130	16	0.020
3.2~5.3	soil	18	170	19	0.150
5.3~7.2	silt	2	170	16	0.025
7.2~8.8	soil silt	2	230	16	0.040
8.8~18.6	soil	32	230	19	0.150
18.6~23.0	silt	4	160	16	0.025
23.0~25.9	sail	7	160	19	0.150
25.9~26.1	clay	31	225	17	0.020
26.1~29.5	soil	31	225	19	0.600
29.5~34.6	gravel	44	440	19	2.000
34.6~35.7	soil	33	440	19	0.600

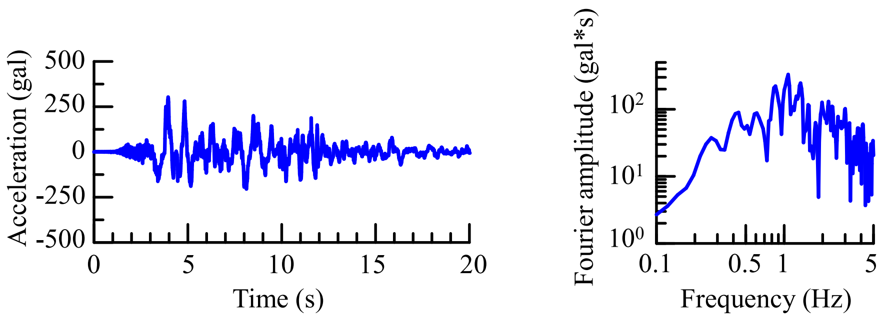


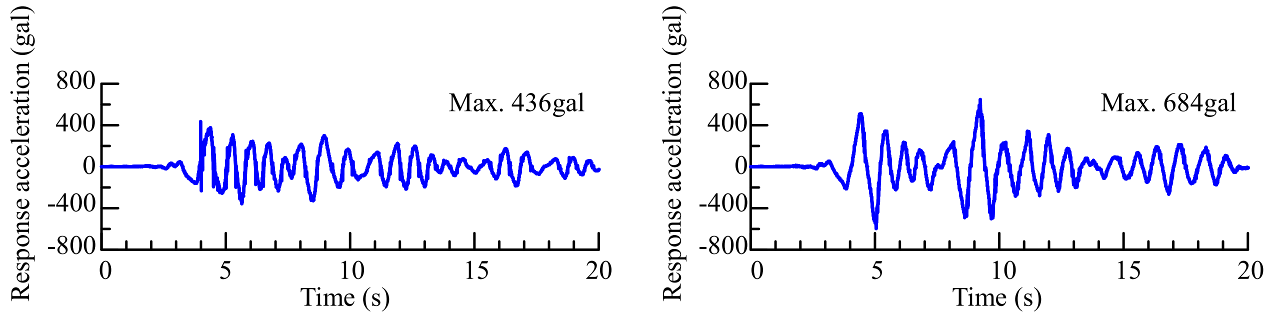
Fig. 5 — Estimated input ground morion (2016 Kumamoto)

The Newmark's β (β =1/4) method was used for numerical simulation. The time step was set to 0.001s. The Rayleigh dumping was set so that the modal dumping ratio were almost constant in the range of about 0.1Hz to 10Hz.

4.2 Comparison of seismic damage

The response acceleration at the top of these structures are shown in Fig. 6. The maximum value are 436gal in structure A and 684gal in structure B as shown in Fig. 6(a) and (b), there is a difference of more than 50%

between the these response values. As for the difference, the relationship between the natural period of these structures and the predominant period of the input ground motion is shown in Fig. 7. The natural periods when the cracks and yielding of columns occur in structure A are 0.51s and 0.80s. On the other hand, the values in structure B are 0.59s and 1.0s. the predominant period of input ground motion is 1.0s, the natural period of structure B is closer to the predominant period of input ground motion than that of structure A. As a result, it seems that the response of structure B is larger than that of structure A.



(a) Structure A Structure B

Fig. 6 ― Response acceleration at the top of structure

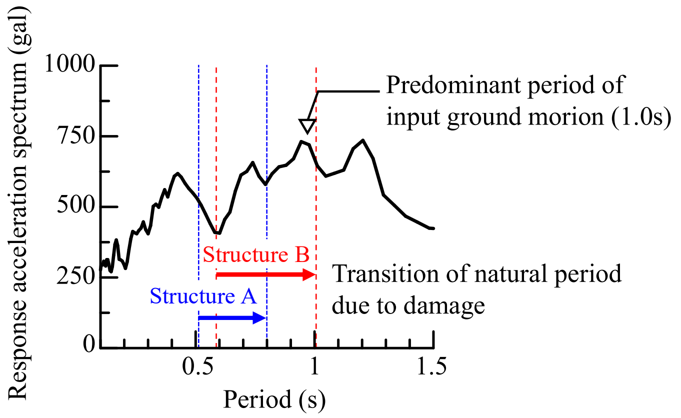
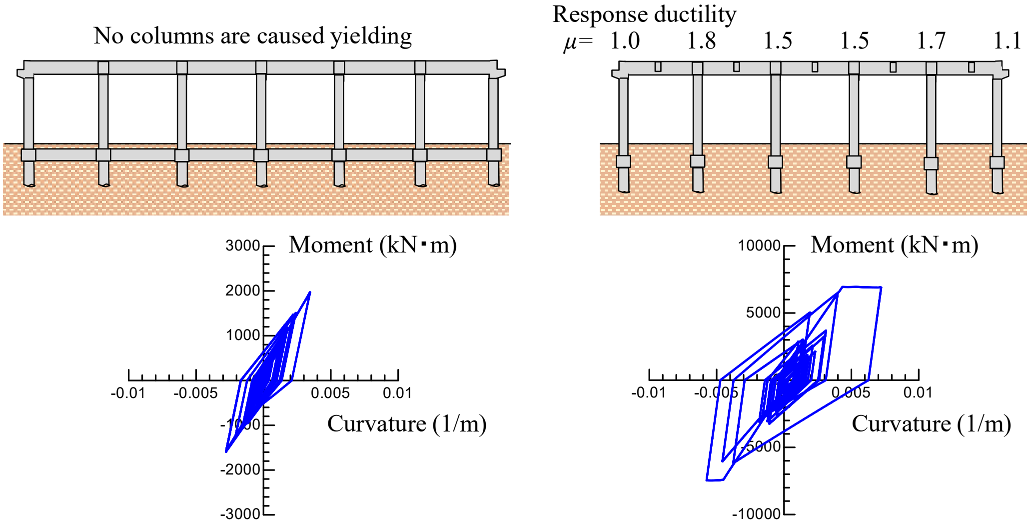


Fig. 7 ― Rerationship between natural period of structures and predominant perid of ground morion

The damage state of structures and the response moment-curvature relationship at the top of column where the response was max are shown in Fig. 8. In the structure A, all the columns were not caused the yielding of the axial rebar. On the other hand, in the structure B in which greater response acceleration occurs, all the columns were caused the yielding of the axial rebar at the top of columns as shown Fig. 8 (b). In particular, the maximum response ductility of columns was 1.8.

Therefore, the responses of these structures are clearly different, and that difference is consistent with the magnitude relationship of damage of the structures which were confirmed after the Kumamoto earthquake.

Consequently, it seems that the consistency of the natural period of structures and predominant period of input ground motion is the factor of the difference in the degree of damage. These strength of structures are almost same, even so, it seems that the response of the structure B which natural period is closer to the predominant period of input ground motion was greater than that of structure A.



(a) Structure A (b) Structure B

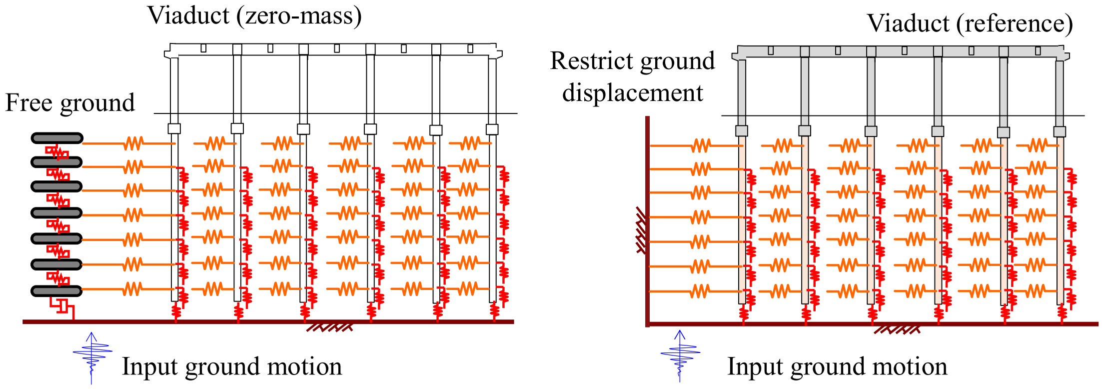
Fig. 8 — Damage state and response moment-curvature relationship

5. Evaluation of ground displacement and inertial force focused in underground beam

5.1 Analytical Model

Usami and Murono[5] showed that the response moment due to the ground displacement reaches the column in the rigid frame viaduct without underground beams. Therefore, it is possibility that the response of these structures also were affected by presence or absence of underground beams. Then, the ground displacement and compare the effects of ground displacement on the response of these structures during an earthquake are focused on.

First, in order to separate and evaluate the effects of ground displacement and inertia on the response of structures, in addition to the reference model (as shown in Fig. 2, hereinafter referred to as the whole system model), a model that does not generate the inertia by setting mass to zero relative to the reference model (hereinafter referred to as the ground system model) was constructed. Furthermore, another model that does not generate the ground displacement by restricting the ground displacements relative to the reference model (hereinafter referred to as the inertial system model) was constructed. These models are shown in Fig.9.



(a) Ground system model (b) Inertial system model

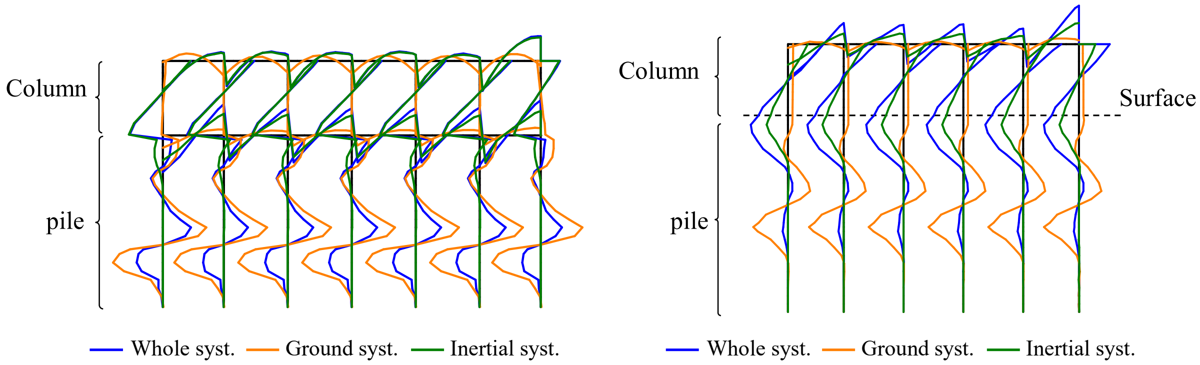
Fig. 9 — Analytical model (ground system model and inertial system model)

The response of the whole system model is included of the effects of the ground displacements and the inertia. However, this study considers the nonlinearity of members and ground, therefore, the response value of the whole system model does not match to the total value of the ground system model and the inertial system model.

5.2 Seismic evaluation under the Kumamoto earthquake

The maximum response moment distributions of the whole system model, the ground system model and the inertial system model in the structure A and B under the input ground motions are shown in Fig. 10.

First, in comparison the distribution of the response moment of columns and piles in the ground system models (blue line in Fig. 10), the response moment of structure A has a distribution only from the piles to the underground beams. On the other hand, that of structure B has a wide distribution from piles to the top of columns.



(a) Structure A (b) Structure B

Fig. 10 — maximum response moment distribution

Secondly, in comparison the results of the inertial system models of these structures (green line in Fig. 10), it can be seen that the response moment of structure A has a distribution only from the top of columns to the underground beams and that of structure B has a continual distribution from the top of columns to the piles.

Therefore, it seems that the response moment caused by ground displacement or inertia is continuously reached from the piles to the columns. As a result, it can be considered that another factor that the response of structure B increased than that of structure A was that the response moment caused by ground displacement was continuously reached the top of columns.

5.3 Seismic evaluation under the design earthquake

A more general study using design ground motion is given. The level 2 design ground motion (spectrum II, ground type G1) was set to the input motion. the Fourier amplitude of the input motion is mostly constant in the range of about 0.1Hz to 10Hz, and the influence of the periodic characteristics of the ground motion on the response of structures can be reduced. In addition, in order to investigate the relationship between the magnitude of input ground motion of the ground and the response of the structure, the maximum acceleration of input ground motion (hereinafter referred to as input acceleration) was adjusted from 100gal to 600gal in 100 gal increments and dynamic analysis was conducted by using each input ground motion.

First, The relationship between the input acceleration and the maximum value of response curvature at the top of columns is shown in Fig. 11. As shown in Fig. 11, in the inertial system model, the increase of the response with increasing input acceleration in these structures are almost same, on the other hand, in the ground system model, the increase of response with increasing input acceleration in the structure B is greater than that increase in the structure A. As a result, in the whole system model, the values of input acceleration when the response curvature exceeds the yielding curvature is 500gal in structure A and 300gal in structure B.

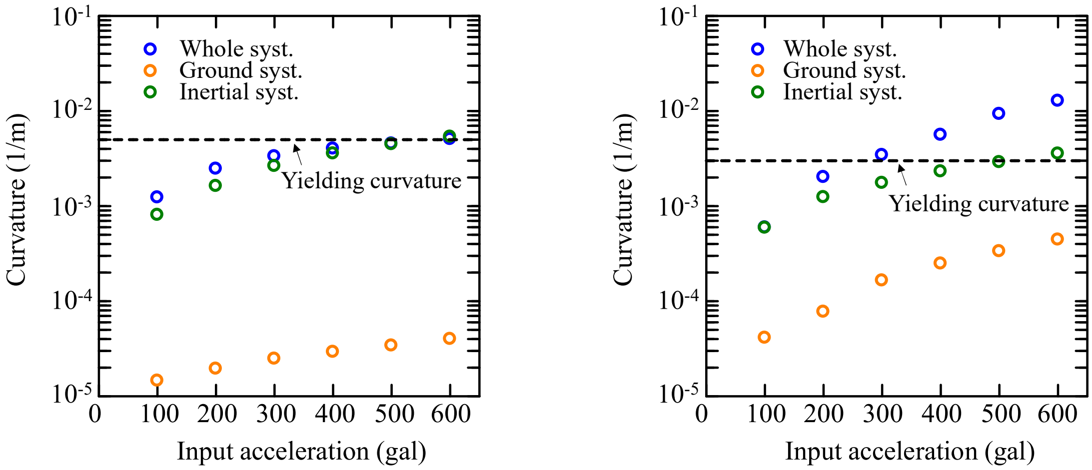


Fig. 11 — Relationship between input acceleration and response curvature

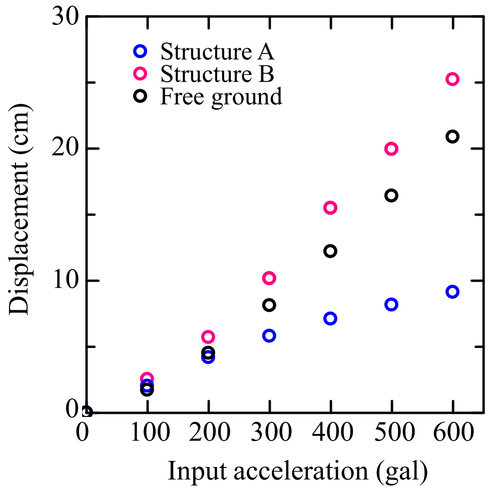


Fig. 12 — Relationship between input acceleration and rerative displacement

Secondly, for comparison focusing on the response of the columns, the relationship between input acceleration and the maximum relative displacement of the top of column and the top of pile for the whole system model is shown in Fig. 12. The figure also shows the maximum surface ground displacement of the free ground. As shown in Fig. 12, the increase of relative displacement with increasing input acceleration in structure B is greater than that increase in structure A.

In addition, the maximum surface ground displacement also increases with increasing input acceleration in both structures. In other words, when the input acceleration is large, the ground nonlinearity also progresses, so it seems that the effect of the ground displacement on the response of the columns in the structure B without underground beams becomes even greater.

6. Conclusion

In this paper, the response of two railway rigid frame viaducts, which differed in the degree of damage in the 2016 Kumamoto earthquake were evaluated. First, the analytical models of these structures were constructed and the dynamic response analysis under the Kumamoto earthquake were conducted, and the factor that caused a difference in the degree of damage of these structures were investigated. Secondly, the effects of the ground displacement and inertia on the response of structures by presence or absence of underground beams were evaluated.

In conclusion, it was clarified that one factor that caused a difference in the degree of damage of these structures in the Kumamoto earthquake was that the natural period of structure without underground beams was closer to the predominant period of the ground motion than that of the other structure with underground beams. Furthermore, the effect of the ground displacement on the response of columns of structure without underground beams was another factor.

However, it should be noted that the consistency of periodic characteristics of the structure and the ground motion was a distinctive case of the Kumamoto earthquake.

7. References

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